REPORT OF GEOTECHNICAL CONSULTATION

PAINE FIELD PASSENGER TERMINAL
SNOHOMISH COUNTY AIRPORT

Prepared for:

Propeller Airports Paine Field, LLC
Everett, Washington

June 1, 2016

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June 1, 2016

Mr. Mark Reichin
Propeller Airports Paine Field LLC
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Report of Geotechnical Consultation
Paine Field Passenger Terminal
Snohomish County Airport
Everett, Washington
URS Project No. 60490227

Dear Mr. Reichin

AECOM submits herewith our draft report of geotechnical consultation for the proposed Paine Field Passenger Terminal to be located in Everett, Washington. The report provides recommendations for design and construction of the proposed facility based on a review of previous geotechnical investigations performed in the vicinity.

Thank you for the opportunity to provide assistance on this project. Please call us if you have questions or need additional information.

Sincerely,
AECOM

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PAINE FIELD PASSENGER TERMINAL
EVERETT, WASHINGTON

1.0 INTRODUCTION

The report presents recommendations for design and construction of the proposed Paine Field Passenger Terminal located near the existing terminal building, at 3220 100th Street SW, Everett, Washington based on the previous subsurface explorations performed by AECOM and others at or near the site. A Site Vicinity Map is presented on Figure 1.

2.0 PROPOSED CONSTRUCTION

The Paine Field Passenger Terminal project includes constructing a passenger terminal expansion building, reconfiguring a passenger parking lot and installing associated utilities near the existing terminal building within the Paine Field property by Propeller Aviation. Propeller Aviation is a tenant at Paine Field, located in unincorporated Snohomish County, south of State Route 526 and east of State Route 525. The project site is just south of 100th Street SW near the existing terminal building, at 3220 100th Street SW, Everett, Washington. The site is located in Township 28N Range 4E, Section 15.

The proposed passenger terminal expansion building would total approximately 29,300 square feet of interior space. The main components of the building would include the entrance and check-in, Transportation Security Administration (TSA) security screening, passenger waiting, boarding area, concessions, baggage handling and claim. The building is a tall single-story building with two large hallways. The building is approximately 280-foot by 160-foot and the overall height of the building is expected to be approximately 28 feet. No basement or other below-ground component is anticipated within the building. The outline of the proposed facility is shown on Figure 2.

It is expected that individual spread footings will be used to support the building. Anticipated column loads were not available for consideration in preparing this report, but AECOM anticipates that exterior and interior column loads will likely range from approximately 150 to 250 kips. Footing bottoms are expected to be at least 2 feet below the ground surface.

The project also includes reconfiguring a passenger parking lot and installing associated utilities near the existing terminal building within the Paine Field property. The project includes four existing parking lots: Areas P1, P2, P3, and P4. Area P1 is on the north of the proposed building. Area P2 is on the east of the proposed building, Area P3 is on the north of Area P1 and Area P4 is on the southeast side of Area P2. Areas P2,P3 and P4 are currently paved areas within the existing terminal area. The project will also include new and/or replacement utilities, including pipes, utilidors, and tanks/vaults, as well as pavement at various locations around the building footprint and parking lots.

3.0 SCOPE OF SERVICES

The scope of services for this geotechnical consultation included the following:

1. Review the available geotechnical information obtained from previous geotechnical investigations performed in the vicinity of the project;
2. Perform a pilot infiltration test to determine the infiltration rate of the soil and to determine feasibility of construction of infiltration facilities;
3. Perform geotechnical analyses to assess options for shallow foundation support, including estimates of foundation settlements; and
4. Prepare a report that presents the estimated soil profile, the results of the pilot infiltration test, and recommendations for design and construction of the building and associated property development components.

4.0 SITE CONDITIONS

4.1 PREVIOUS INVESTIGATIONS

Previous geotechnical investigations at the project location and the general vicinity have included the following:

1. URS Corporation, "Draft Runway 16R-34L Rehabilitation and Related Work, Report of Geotechnical Investigation, Snohomish County Airport, Everett, Washington", March 27, 2009 (URS, 2009).
2. GeoEngineers - Construction Observations during Paine Field Tower Apron, Nose Gear Tie Down Anchors, November 2010 (GeoEngineers, 2010).
3. GeoEngineers, "Geotechnical Engineering Services, Snohomish County Airport/Paine Field General Aviation/Corporate Ramp Expansion Project, Everett, Washington", February 21, 2005 (GeoEngineers, 2005).

Borings and test pits that were advanced and excavated as described by GeoEngineers (2005) were in the immediate vicinity of the proposed passenger terminal expansion building. The boring and test pit locations that are in the immediate vicinity of the proposed building are shown on Figure 2. The logs of these borings and test pits are included as Appendix A.

No new borings were advanced for this project.

4.2 SURFACE CONDITIONS

The existing site is generally relatively flat with an average elevation ranging from approximately 590 to 600 North American Vertical Datum (NAVD) 88, and slopes from east to west at an average slope of one percent.

4.3 GEOLOGIC SETTING

Geologic maps indicate that the site lies within an area occupied by deposits of glacially deposited Vashon till (Qvt), with nearby deposits of advance glacial outwash (Qva) which also underlies the glacial till. The geologic publications also indicate that liquefaction potential for surface soils are categorized as “very low to low” in the most recent liquefaction susceptibility mapping (Palmer et al, 2004).

4.4 SUBSURFACE CONDTIONS

The geotechnical investigation performed by GeoEngineers (2005) was in the immediate vicinity of the project as shown on Figure 2. Based on GeoEngineers (2005) and engineering judgement based on site knowledge, the subsurface exploration information in the immediate vicinity of the project indicates the following soil profile, as illustrated in Figure 3 Idealized Soil Profile:

<u>STRATUM 1 - Medium Dense Silty Sand (Fill)</u>

Fill soil typically included silty sand with occasional gravel as well as occasional charcoal. The consistency of this soil was usually medium dense. The thickness is typically 1.5 feet below ground surface (bgs).

<u>STRATUM 2 - Medium Dense to Dense Silty Sand With Gravel (Weathered Glacial Till)</u>

This soil in this stratum consisted of silty sand with gravel and cobbles. The consistency of the soil is typically medium dense to dense. This layer is typically 2.5 feet thick and is typically encountered between 1.5 and 4 feet bgs.

<u>STRATUM 3 - Dense to Very Dense Silty Sand With Some Gravel (Glacial Till)</u>

This soil in this stratum consisted of silty sand with gravel. The consistency of the soil is typically dense to very dense. This layer is typically encountered at 4 feet bgs. It is considered to have high shear strength and very low compressibility.

Subsurface soil conditions encountered during the anchor tie down installation within the proposed building footprint (GeoEngineers, 2010) indicated similar conditions described above. The soil conditions in the report were described as “Gray silty sand with occasional gravel (fill) overlying dense to very dense silty sand with occasional gravel (glacial till). Dense to very dense glacial till was verified by means of probing with a 1/2-inch diameter steel probe and drilling with a hand auger at a depth of about 4.5 feet’. (GeoEngineers, 2010).

Available information from the previous investigations provided a generally favorable assessment of the suitability of foundation soils, but with a potential for localized pockets of deeper fill. For example, some of the test pits and boring logs of the explorations that are almost 1000 feet west to the proposed building where URS, 2009 geotechnical investigation was performed indicated the presence of fill soil to depths up to 12 feet, including presence of organic matter and organic silt. These fill materials were apparently placed to fill in local ravines. However, much of the fill consisted of medium dense to dense granular materials. Firm glacial soils were shown underlying the fill, where present.

GROUNDWATER

Static groundwater was not observed in the explorations completed in 2005 by GeoEngineers. Perched groundwater was observed in 10 of the 30 test pit explorations and 2 of the 5 borings. The perched groundwater was typically observed near the contact of the fill and weathered glacial till or near the transition between the weathered and the unweathered glacial till. Perched groundwater was also observed within the advance outwash soils below the glacial till in Boring B-4. Perched groundwater levels may be present at times throughout the year and will typically fluctuate as a function of season, precipitation and other factors (GeoEngineers, 2005).

4.5 LABORATORY TESTING

Laboratory testing was not performed on the samples obtained from the explorations performed in the immediate vicinity. However, testing performed on the samples from the explorations that are approximately 1000 feet west to the proposed building show almost 50 percent fines in Stratum 1 Fill layer (URS, 2009).

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GEOTECHNICAL DESIGN PARAMETERS - GENERAL

A summary of estimated soil parameters for each of the soil strata encountered at the site is provided in Table 1, presented after the text of the report. Table 1 is to assist with the design of the proposed facility. The values provided in Table 1 have been estimated using a combination of measured field and laboratory data together with published data on similar soils. It should be noted that in most cases the values listed in Table 1 are intended to represent average or slightly on the conservative side of average conditions. Exceptions are that the values listed for passive earth pressure coefficient and soil-to-concrete friction are considered ultimate values, to which a safety factor of at least 1.5 should be applied to achieve design values. Natural variations in stratigraphy and soil parameters are expected throughout the site, and the values in Table 1 may not be strictly representative of all locations.

5.2 SEISMIC CONSIDERATIONS

The geotechnical-related considerations for seismic design must be in accordance with provisions of the International Building Code (IBC) (ICC, 2012), and the associated American Society of Civil Engineers Standard ASCE/SEI 7-10 (ASCE, 2010). The spectral response accelerations for the "Risk-Targeted Maximum Considered Earthquake" (MCE_R) are obtained from USGS online data (U.S. Geologic Survey, Earthquake Hazards website, 2008 data). The recommended values of acceleration for the spectral response at the Site (47.90903 N Latitude and 122.27905 W Longitude) are presented in Table 2. This table presents the recommended values of pertinent seismic design parameters that may be needed for the project.

The Site Class is selected using the definitions in Chapter 20 of the ASCE 7-10 Standard considering the average properties of soils in the upper 100 feet of the soil profile at the site. The available data indicates that the likelihood of seismic-induced liquefaction at this site is low because of the dense to very dense condition of the subsurface soils, and accordingly a Site Class C (Very Dense Soil and Soft Rock Profile) is appropriate for this project.

Horizontal peak ground acceleration (PGA) values were estimated using the USGS website Hazard Curve Application tool, together with Table 11.8-1 of ASCE 7-10 where a site-specific ground motion study is not performed. The PGA values for the MCE (two percent chance of being exceed in 50 years, or return period of 2,475 years) for the Site Class C category, and for an event having a 10 percent chance of being exceeded in 50 years (i.e., having a return period of 475 years) are listed in Table 2.

TABLE 2 - Recommended Seismic Design Parameters

Parameter	Value	Reference
Soil Profile Site Class	C	2010 ASCE-7 Table 20.3.1
0.2 Second Risk Targeted Maximum Considered Earthquake Spectral Acceleration (S_s)	1.441g	2012 IBC Section 1613.3.3
1.0 Second Risk Targeted Maximum Considered Earthquake Spectral Acceleration (S_l)	0.563g	2012 IBC Section 1613.3.3
Site Coefficient (F_a) for Site Class C	1.0	2012 IBC Table 1613.3.3(1)
Site Coefficient (F_v) for Site Class C	1.3	2012 IBC Table 1613.3.3(2)
0.2 Second Site Class effects adjusted Risk Targeted Maximum Considered Earthquake Spectral Acceleration (S_{Ms}) for Site Class C	1.441g	2012 IBC Equation 16-37
1.0 Second Site Class effects adjusted Risk Targeted Maximum Considered Earthquake Spectral Acceleration (S_{Ml}) for Site Class C	0.563g	2012 IBC Equation 16-38
0.2 Second Site Class effects adjusted Risk Targeted Design Spectral Acceleration (S_{Ds}) for Site Class C	0.961g	2012 IBC Equation 16-39
1.0 Second Site Class effects adjusted Risk Targeted Design Spectral Acceleration (S_{Dl}) for Site Class C	0.488g	2012 IBC Equation 16-40
Horizontal Peak Ground Acceleration PGA MCE (2% in 50 yrs/ 2475 yr return period)	0.630g	2012 IBC/2008 USGS PSH Deaggregation
Horizontal Peak Ground Acceleration PGA (10% in 50 yrs/ 475 yr return period)	0.320g	2012 IBC/2008 USGS PSH Deaggregation

5.3 FOUNDATION SUPPORT - SHALLOW FOOTINGS

The soil conditions encountered are generally considered favorable for building foundation support, with shallow foundations designed for 5000 pounds per square foot (psf) after suitable preparation of fill soils encountered within the building perimeter. The preparation would include compaction of “suitable” Stratum 1 fill soils (granular soil having less than approximately 25 percent fines) using a backhoe-mounted compactor (Hoepac) to minimize the potential for settlement under building loads.

Foundations supported directly on Stratum 2, weathered glacial till and Stratum 3, glacial till should also be designed using the conservative value of allowable bearing pressure of 5000 psf, although higher values are possible. The Geotechnical Engineer will review the data and provide the higher value as needed.

The allowable soil bearing value recommended above may be increased by one-third for transient load conditions from wind or seismic sources.

All footings should be embedded at least 1.5 feet for frost protection. A minimum width of 1.5 feet for wall footings and 2 feet for spread footings is recommended.

SETTLEMENT

Total settlement of footings will depend on the size of the footing and the actual applied pressure. Figure 4 presents the estimated total settlement of square footings as a function of footing width for applied pressures of 5000 psf. For long footings (i.e., having a length of at least 2 times the width) the settlement values in Figure 4 should be increased by a factor of 1.5. Figure 4 has been prepared using the assumption that up to 4 feet of properly prepared Stratum 1 fill or Stratum 2 and Stratum 3 soils will be present immediately beneath the footing. For that reason, the figure shows settlements at first increasing under the primary influence of the fill layer. The total settlement for 2 to 10 feet square footings ranged from 0.07 to 0.11 inches. For larger footings, the underlying more dense and stiffer glacial soils control the settlement, which can be considered to remain steady at less than approximately 0.20 inches.

The settlements are expected to be primarily elastic in nature, occurring essentially simultaneous with the application of load. No substantial long term settlements are anticipated.

FOUNDATION STIFFNESS AND RELATED PARAMETERS

Foundation load-deformation characteristics may be needed for design analyses. The recommended stiffness parameters for the soil at this site are presented in Table 1, and include the static elastic modulus E obtained from AECOM experience and published information on in-situ tests this type of glacial till soil.

Typical values of modulus of subgrade reaction k (also called coefficient of subgrade reaction) in pounds per cubic inch have been provided in Table 1 for design of mats and slabs. The values have been taken from published results of field tests as well as consideration of guidance in American Concrete Institute publications.

AECOM recommends that equivalent spring constants for vertical translation, horizontal translation, torsion and rocking modes be estimated using the relationships presented in Figure 4-4 of ASCE 41-06 (ASCE, 2007). This publication presents corrections for footing shape and embedment depth. Soil parameters needed for these relationships are provided in Table 1 of this report.

SUBGRADE PREPARATION

Footing subgrades should be cleared of loosened soils and protected against ponding of surface water which may cause deterioration and possible settlement under the allowable pressures to be applied by the future footings. If the subgrade surface softens due to exposure to surface water and subsequent foot traffic during placement of reinforcing steel, the softened material should be removed to the maximum possible extent prior to pouring of concrete. The preferred method of protecting the subgrades during wet weather construction is to place a thin layer of lean concrete or Controlled Low Strength Material (CLSM) or Controlled Density Fill (CDF) as indicated in Section 5.10 Earthwork and Construction Concerns. A layer of well graded crushed stone may be used for protection, but should be limited to less than 1 foot in thickness. This fill should be compacted to at least 95 percent of the maximum dry density as measured using the Standard Test Methods for Laboratory Compactive Characteristics Using Modified Effort (ASTM D-1557).

If existing unsuitable material (silt, clay, organics, construction debris) encountered at the footing subgrade level in Stratum 1 fill material, the fill material should be overexcavated until suitable Stratum 1 granular fill or native Stratum 2 glacial till is encountered. The overexcavated soil should be replaced with on-site or imported structural fill consisting of well graded sand and gravel that is compacted to 95 percent of the maximum dry density as measured using ASTMD-

1557. The fill should have a maximum particle size of approximately 2 inches, and it should have less than approximately 7 percent fines. Another acceptable option is to replace the overexcavated material with CDF having a minimum compressive strength of 100 psi. Before placing the imported fill, the exposed subgrade should be evaluated by the Geotechnical Engineer, who may require that additional overexcavation occur depending on the condition of the exposed subgrade soil.

Existing suitable Stratum 1 fill (silty sand and gravel) encountered at the footing subgrade level should be surface compacted with a backhoe-mounted vibratory compaction tool (e.g. Hoepac) to provide a firm and unyielding subgrade, as confirmed by the Geotechnical Engineer.

5.4 FLOOR SLABS

Floor slabs may be supported on grade on properly prepared suitable Stratum 1 fill material or Stratum 2 and 3 or on properly compacted granular structural fill added during construction. The existing fill could include the sand and gravel base course apparently used for the existing pavement.

The slab should be underlain by a base course of at least 4 inches of free draining sandy gravel or crushed stone with less than 5 percent fines. In addition, in areas to be occupied by potentially moisture sensitive office space, the slab should be provided with a plastic vapor barrier to prevent moisture migration to the building interior. The native glacial till is very poorly drained and will create perched water conditions where surface water is not captured by footing drains. An underslab drainage system is not considered necessary for this building, as further discussed in Section 5.8 Dewatering and Drainage.

Recommended values of vertical modulus of subgrade reaction are presented in Table 1.

Settlement of the floor slabs is expected to be negligible for applied pressures less than about 200 psf.

Prior to its use as the subgrade for the floor slab, the existing fill should be proofrolled to densify loose or disturbed zones. Precautions related to wet weather deterioration are discussed in Section 5.10 Earthwork and Construction Concerns.

If imported structural fill is used to support the slab, the fill should be compacted in 6- to 8-inch lifts to 95 percent of the maximum dry density as measured using ASTM D-1557. The soil should be compacted at moisture content values within two percentage points of optimum, as measured using ASTM D-1557.

5.5 LATERAL EARTH PRESSURES

Lateral earth pressures are required for estimating the resistance to lateral loads on foundations, and for designing minor permanent subsurface walls that may be needed for sumps, vaults or similar shallow pits. The recommended values of earth pressure coefficients and soil unit weight for estimating lateral earth pressures are presented in Table 1. Lateral earth pressures should be estimated assuming a uniform increase with depth similar to an equivalent fluid. The equivalent fluid unit weight is obtained by taking the product of the total unit weight of the soil times the appropriate earth pressure coefficient presented in Table 1.

The active case usually applies to walls that are permitted to rotate or translate inward (i.e. away from the retained soil) by approximately 0.002H, where H is the height of the wall. This case would be appropriate for a retaining wall or an abutment founded on soil. The at-rest case applies to unyielding walls, such as a rigidly connected basement wall. The passive value includes a factor of safety of 1.5. All these values apply to the case of drained soil conditions where water pressures are not permitted to develop, and should be accompanied by adequate drainage measures.

During an earthquake the active and at-rest pressures will temporarily increase and the passive pressures will temporarily decrease, as described by Whitman (1990) using the Mononobe Okabe equation and by Sherif et al (1982) using bench-scale experimentation. However, considering the competent soils at this site and the expectation of shallow footings and limited subsurface walls, the seismic effects on lateral earth pressures can be ignored.

Lateral load resistance may also be provided by soil-concrete friction along the bottom of footings and mats. The recommended soil-concrete friction coefficients for the native and possible granular structural fill soils are presented in Table 1. The soil-concrete friction may be used in combination with the passive lateral earth pressure acting against the side of the footing or mat.

5.6 PAVEMENTS

The properly prepared onsite fill soil and the native glacial till are expected to provide suitable subgrades for pavements, with estimated California Bearing Ratio (CBR) values of at least 15. Both the fill and the native glacial till soil are expected to be poorly drained due to the presence of approximately 20 to 25 percent or more fines (silt and clay).

Asphalt or concrete pavement may be used, underlain by either untreated aggregate or asphalt treated base placed on the compacted subgrade. The crushed untreated aggregate base course should conform to the characteristics listed in Section 9-03.9(3) of the current WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT, 2016). The minimum recommended thickness of untreated aggregate is 4 inches, and the minimum thickness of asphalt treated base is 3 inches. The total thickness of the pavement system should be selected based on the level of traffic anticipated.

Pavement subgrades should be compacted and repaired as necessary by removing zones of unfavorable materials such as wet or soft fines, construction debris, or wet surficial glacial till. The proofrolling should consist of at least two passes of a vibratory roller having an operating weight of at least 10 tons. The compacted surface should be firm and unyielding, with no discernable deflection under the roller load.

5.7 EXCAVATIONS

The Stratum 1 silty sand with fill can be considered a Type B soil from the standpoint of Occupational Safety and Health Administration (OSHA)/Washington State Safety and Health Administration (WSHA) regulations for excavations, trenching and shoring. This means that temporary cuts greater than 4 feet deep should be sloped at no steeper than one 1 horizontal to one vertical (1H:1V).

The native Stratum 2 and 3 (weathered glacial till and glacial till) can be considered a Type A soil from the standpoint of OSHA/WSHA regulations for excavations, trenching and shoring. This means that temporary cuts greater than 4-feet deep should be sloped at no steeper than 0.75 horizontal to1 vertical (0.75H:1V).

Substantial excavations are not anticipated for this project. Excavations may be open cut or supported using an appropriate shoring system. Possible shoring solutions for localized excavations deeper than 4 feet include trench boxes or traditional cantilever soldier piles with wood lagging. Lateral earth pressures applied to shoring systems should be estimated as indicated in Section 5.5.

5.8 DEWATERING AND DRAINAGE

During construction the occasional zones of free draining sand and more permeable granular zones within the Stratum 1 fill and Stratum 2 glacial till soil may yield small quantities of perched water that will flow into the excavation. If encountered, this drainage is expected to be controlled by sumps and pumping.

If any structure is embedded in Stratum 2 and 3, the structure may experience uplift pressures because of the low permeability of the materials. Therefore, providing drainage using sump pumps or piping system may need to be considered.

5.9 INFILTRATION RATES

A pilot infiltration test (PIT) was performed on May 19, 2016 at a proposed infiltration facility, which is located to the north of the proposed building and to the east side of the proposed wet vault required for stormwater treatment at the site. The PIT was performed in accordance with the Washington Department of Ecology (WDOE), Stormwater Management Manual for Western Washington (2012, amended on December 2014) to determine the capability of site soils to infiltrate the stormwater.

The location of the PIT is shown on Figure 2. A test pit was excavated at the PIT location and the following subsurface conditions were encountered at the test pit:

- 0 to 1 foot depth bgs: Fill (Stratum 1)
- 1 to 2 foot depth bgs: Weathered glacial till (Stratum 2)
- 2 to 3 foot depth bgs: Glacial till (Stratum 3)

The length, width and depth of the test pit were 6, 3 and 3 feet, respectively. The corners of the test pit were squared up by hand with a shovel. The test pit was filled with water and the test pit walls were allowed to soak for six hours prior to collecting the water level measurements. Water was filled in the test pit to 2-foot bgs for the PIT. A measuring rod was installed firmly in the center of the test pit for water level measurements.

The water level measurements were started in the test pit after six hours of soaking and were recorded every 15 minutes for one hour. Water remaining in the test pit was discharged to a location onsite. The test pit was backfilled and compacted with excavated soil.

The infiltration rate calculated for the existing soils at the PIT location was approximately 0.04 inches per hour before any correction factors were applied. The infiltration rate after the correction factors were applied was 0.02 inches per hour, which is considered too low for constructing an infiltration facility.

Additional information gathered during the PIT and correction factors used for calculating the infiltration rate are provided in Appendix B.

5.10 EARTHWORK AND CONSTRUCTION CONCERNS

The existing pavement base course and the “suitable” Stratum 1 on site soils are both considered re-usable as compacted structural fill or for general grading, with the understanding that the reasonably high percentage of fines makes both soils vulnerable to deterioration during wet weather construction. In a disturbed condition, Stratum 2 and Stratum 3 glacial till are vulnerable to moisture-induced deterioration, and re-use will be difficult to impossible except in periods of more favorable weather. During the winter months, at least 50 to 75 percent of the excavated soil is expected to become too wet for re-use on site. Even the surface of the exposed but undisturbed till may soften to a depth of several inches, or become disturbed to greater depths by foot and construction vehicle traffic. This softened material should be removed from subgrades of footings, slabs and pavements prior to concrete placement. Protection of freshly exposed glacial till subgrade surfaces can be accomplished by covering the surface within a thin layer of lean concrete or CLSM or CDF having an unconfined compressive strength of at least 100 psi and meeting the requirements of Section 2-09.3 (1) E of the WSDOT standard specifications

During recompaction of the Stratum 1 fill, cobbles should be removed or spread out within the soil such that each cobble is surrounded within a distance of at least 6-inches by silty sand and gravel rather than other cobbles. This restriction is to promote a more even compaction density, and to avoid the possible formation of voids around nested cobbles. Particles larger than 2 inches in diameter should not be placed directly in contact with floor slabs or walls in order to avoid stress concentrations in these structural members.

Onsite or imported structural fill may be used to support footings, floor slabs and pavements. Structural fill soil should be compacted in 6 to 8-inch lifts to 95 percent of the maximum dry density as measured using ASTM D-1557. The compaction moisture content should be within 2 percent of the optimum value per ASTM. Backfill in utility trenches should be compacted to 90 percent of the maximum dry density (ASTM D-1557) where the utility trench is not directly beneath pavement. Fill for general grading purposes may be compacted to 88 percent of the maximum dry density (ASTM D-1557), except on slopes steeper than 3H:1V where 93 percent should be achieved.

Imported structural fill should consist of well graded granular soil such as Washington State Department of Transportation Section 9-03.12 Gravel Backfill (WSDOT 2016) or other well graded natural or crushed products.

In a disturbed condition, Stratum 2 and Stratum 3 glacial till are expected to be moderately to highly erodible. Erosion control efforts during construction should be adopted and best management practices applied as necessary including stabilization of construction entrances with crushed rock or quarry spalls; mulching of exposed surfaces; protecting catch basins by wrapping grates in geotextile, or by surrounding the basin with straw bales wattles, check dams; and/or silt fencing.

6.0 LIMITATIONS

The recommendations and descriptions presented in this report are based on soil conditions disclosed by the borings drilled and test pits excavated during previous investigations at the site. The existing subsurface information referred to herein does not constitute a direct or implied warranty that the soil conditions between exploration locations can be directly interpolated or extrapolated, or that subsurface conditions and soil variations different from those disclosed by the borings will not be revealed. If, during construction, subsurface conditions different from those described herein are observed, such conditions should be reviewed and the recommendations given herein revised as necessary. Similarly, changes to the structure including modified load magnitudes and footing sizes should be brought to our attention so that the potential effect of these changes can be assessed.

7.0 REFERENCES

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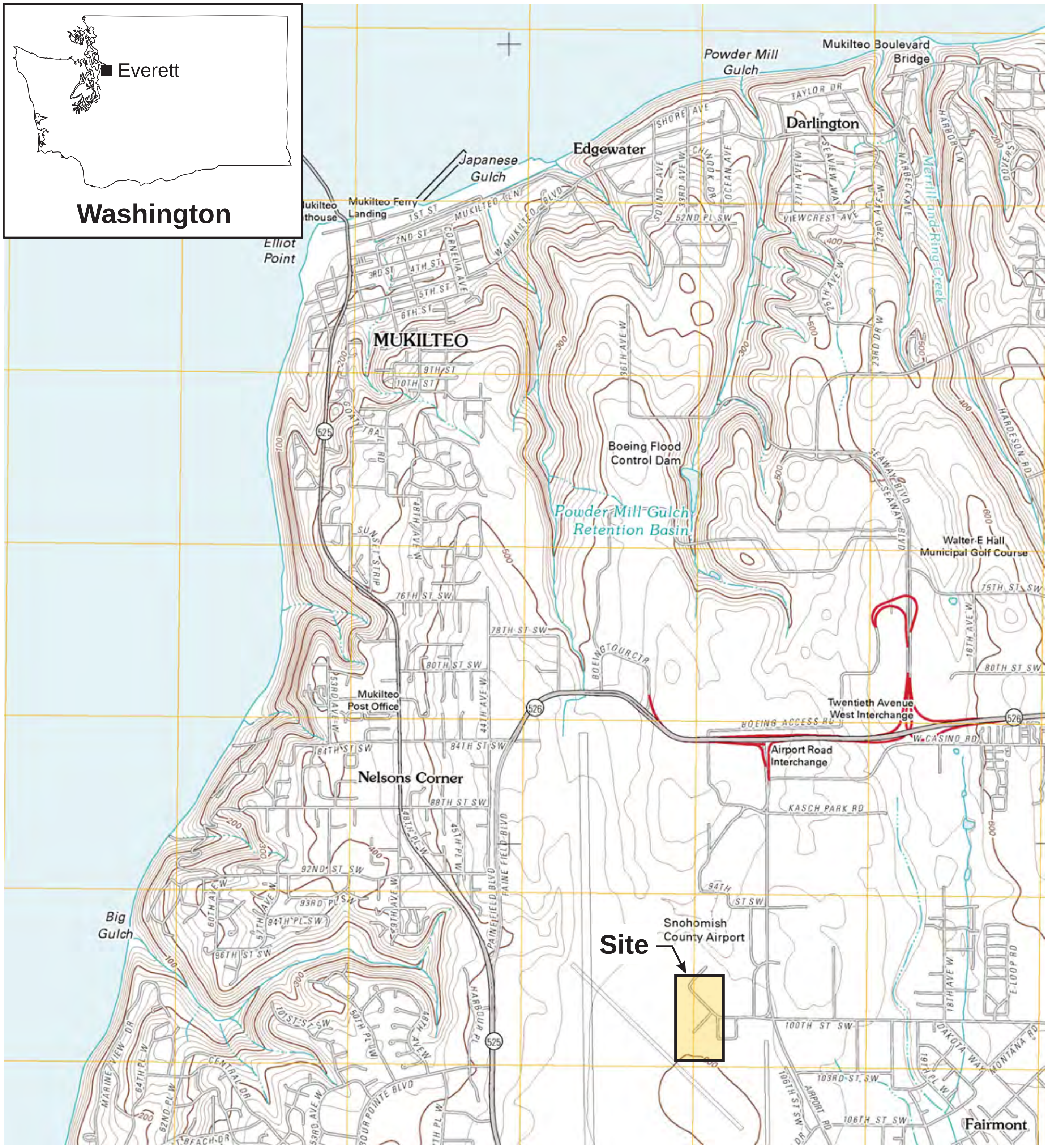
TABLE 1 - Summary of Recommended Soil Parameters for Paine Field Passenger Terminal Project

		Stratum 1	Stratum 2	Stratum 3 [4]
	Compacted	Medium Dense Silty Sand	Medium Dense to Dense Silty Sand with Gravel	Dense to Very Dense Silty Sand with Some Gravel
ITEM	Existing Fill	Fill	Weathered Glacial Till	Glacial Till
Total Unit Weight γ (pcf)	125	120	130	130
Friction Angle ϕ (degrees)	36	34	40	40
Cohesion c (psf)	0	0	200	500
Drained Friction Angle ϕ' (degrees)	36	34	40	40
Static Elastic Modulus E (ksf)	1200	800	10000	10000
Poisson's Ratio v	0.32	0.3	0.35	0.35
Active Earth Pressure Coeff Ka	0.26	0.28	0.22	0.22
At-Rest Earth Pressure Coeff Ko	0.41	0.44	0.36	0.36
Passive Earth Pressure Coeff Kp	3.85	3.54	4.60	4.60
Seismic Active Earth Pressure, Kae	0.94	1.07	0.79	0.79
Seismic Passive Earth Pressure, Kpe	2.25	1.90	2.97	2.97
Soil-Concrete Friction Coeff.	0.5	0.45	0.45	0.45
Modulus Subgrade Reaction ks (pci)	300	300	400	400
Soil Spring Coefficient (pci)	300	300	400	400
Shear Wave Velocity (ft/sec)	-	-	1500	1500

NOTES:

1. The ks values are typical for results of tests on 30-inch diameter plate, and need not be corrected for size or shape of loaded area.
2. Values listed above generally represent average to the slightly conservative side of average values based on interpretation of available data. Natural variability of soil conditions and parameters are expected to occur throughout the site. An exception is that the values of Kp and Soil-Concrete Friction Coefficient are considered "Ultimate" values which should be divided by a safety factor of at least 1.5 .
3. The static E values apply to moderately large shear strain levels of approx 10^{-1} percent, i.e. for footing loads.
4. Lower bound values were selected for glacial till.

FIGURES



Source: USGS 7.5-minute topographic quadrangle, Mukilteo, Washington, 2011

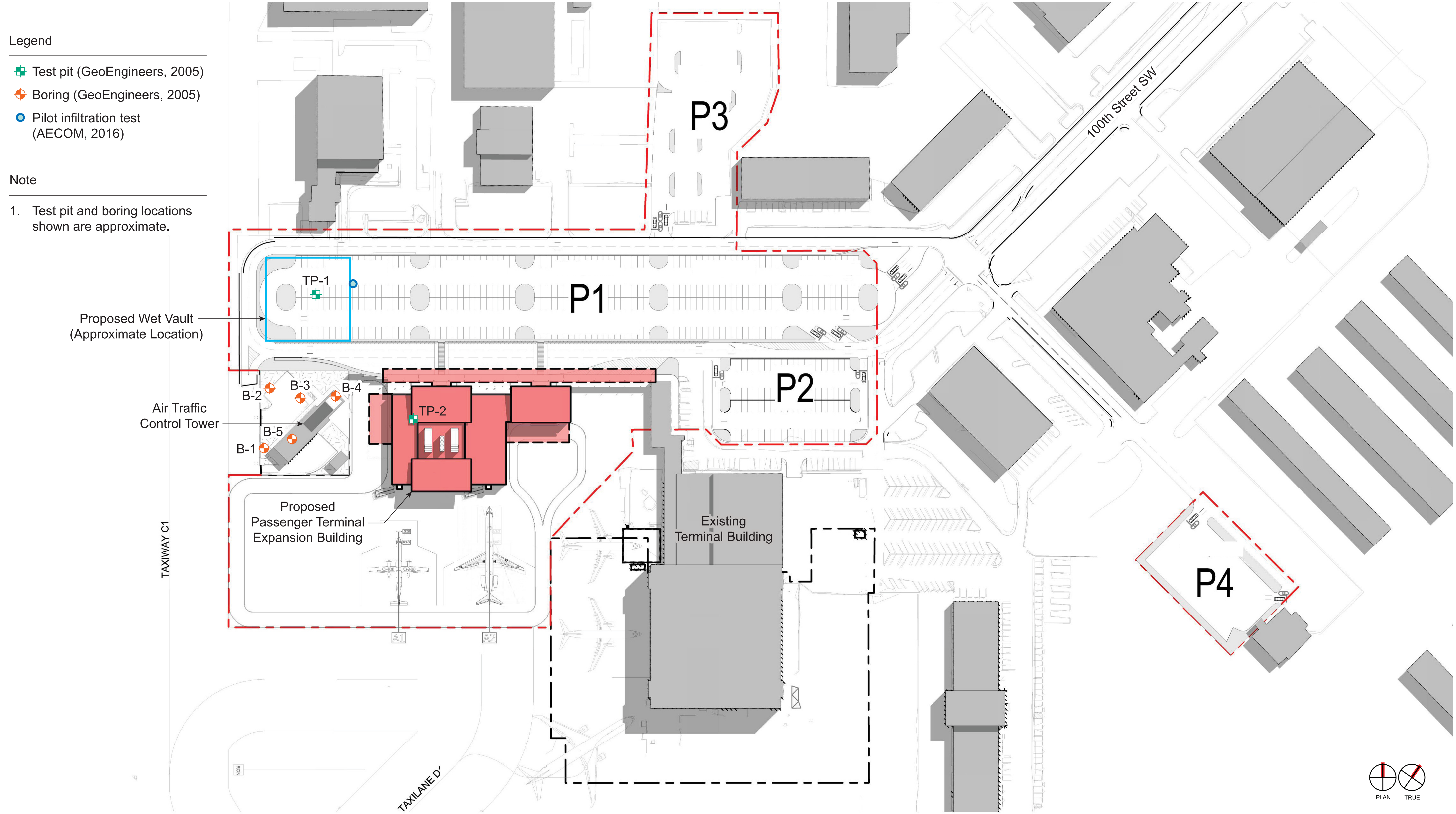
0 2,000 4,000
Scale in Feet

Figure 1
Site Vicinity Map

Job No. 60490227

AECOM

Paine Field Passenger Terminal
Snohomish County Airport, Washington



60490227_05.ai

Figure Source: Fentress Architects, SEPA Checklist Figure 3A, March 11, 2016

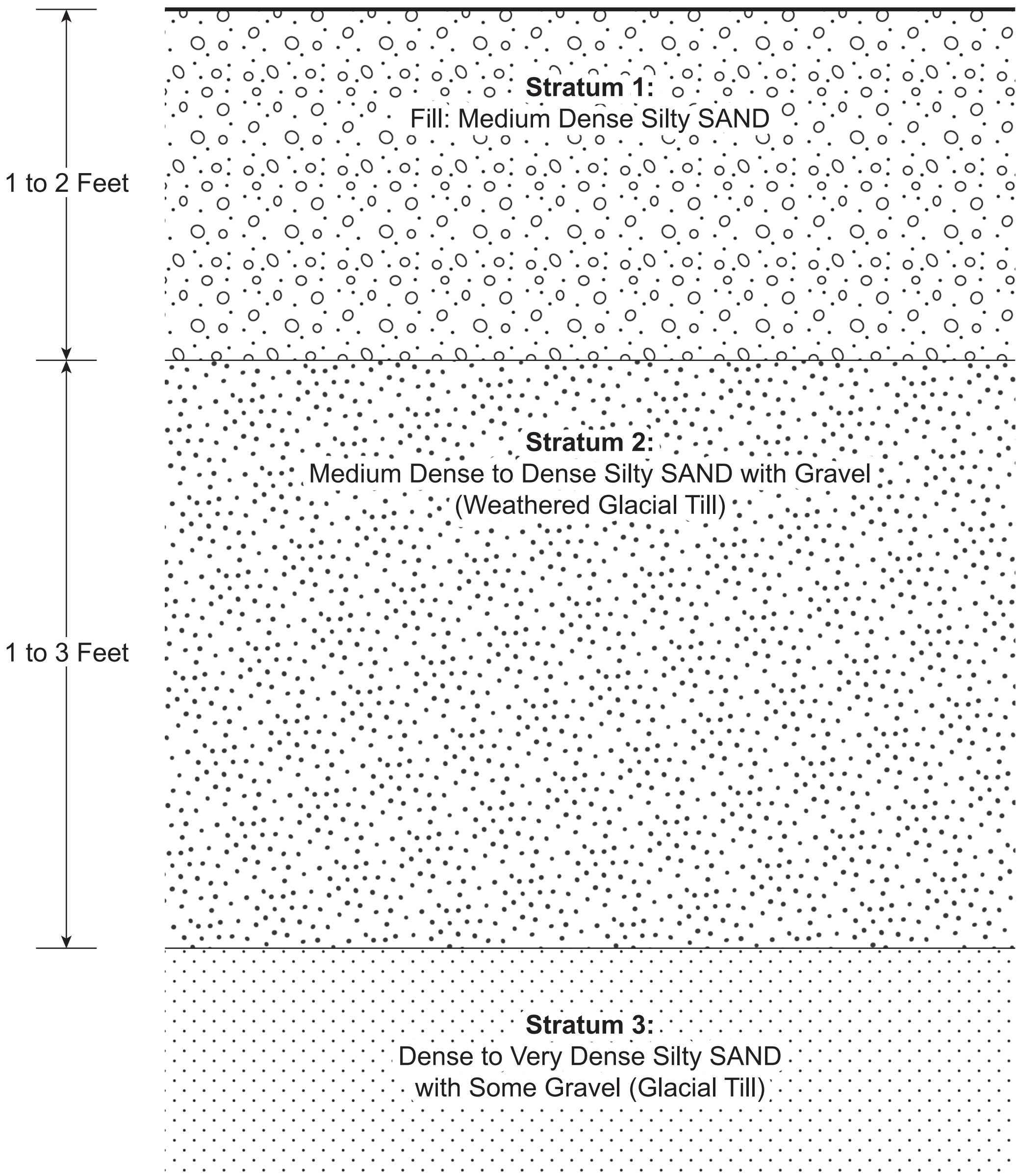
Job No. 60490227

Figure 2
Site Plan

AECOM

Paine Field Passenger Terminal
Snohomish County Airport, Washington

60490227_03.ai



Note

1. Based on test pits and borings at site and in vicinity. Actual thickness of the strata may vary.

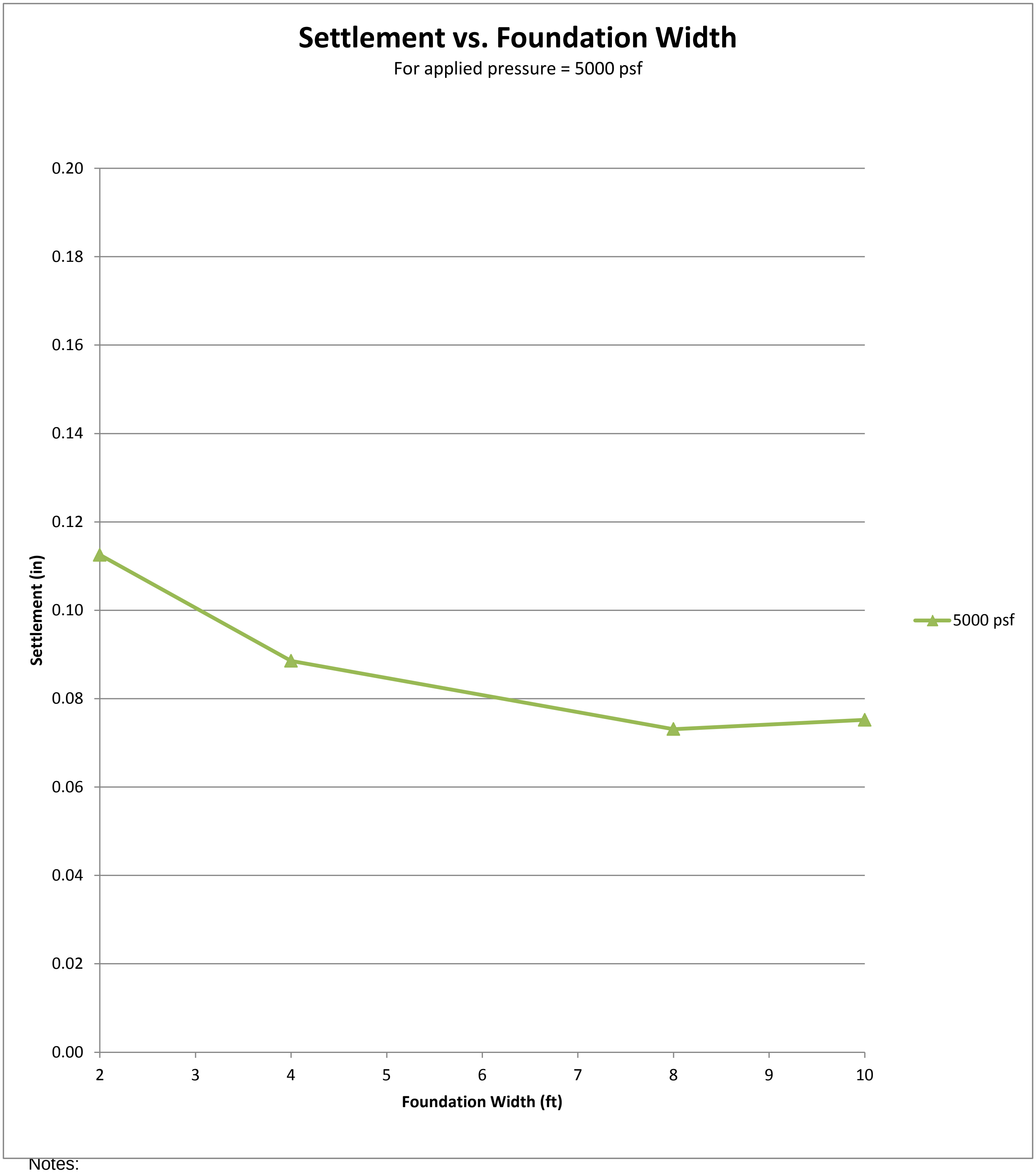
Figure 3

Idealized Soil Profile

Job No. 60490227

AECOM

Paine Field Passenger Terminal
Snohomish County Airport, Washington



Notes:

1. Assumed rigid footing.
2. Assumed the bottom of footing 2 feet below ground surface.

Figure 4

Job No.: 60490227

Settlements of Shallow Foundations vs. Footing Sizes

Paine Field Passenger Terminal
Everett, WA

Settlement Schmertmann new-18MAY16.xlsx

APPENDICES

APPENDIX A

BORING AND TEST PIT LOGS

Date Excavated: 01/17/05

Logged by: ABA

Equipment: Case 580 Backhoe

Surface Elevation (ft): Approximately 594

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				TS	Topsoil and root zone		
					SM	Grayish brown with oxidation staining silty fine to medium sand with gravel and charcoal (medium dense, wet) (fill)		
			1		SM	Grayish brown silty fine to medium sand with gravel and cobbles (weathered till) (dense to very dense, moist)	13	
			2					
					SM	Grades to gray (very dense, moist) (glacial till)		
			3					

Test pit completed at 3½ feet on 01/17/05
Slow groundwater seepage observed at 1 foot
No caving observed
Refusal at 3½ feet

5

10

Note: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

V6_GTTPIT P:\5\5530003\00\FINALS\553000300.GPJ GEIV6_1.GDT 2/11/05

LOG OF TEST PIT TP-1

GeoEngineers

Project: Paine Field Ramp Expansion
Project Location: Everett, Washington
Project Number: 5530-003-00

Figure A-2
Sheet 1 of 1

Date Excavated: 01/17/05

Logged by: ABA

Equipment: Case 580 Backhoe

Surface Elevation (ft): Approximately 595

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				TS	Topsoil		
					SM	Grayish brown with oxidation staining silty fine to medium sand with gravel (medium dense to dense, moist) (fill)		
		X	1		SM	Brownish gray silty fine to medium sand with gravel and cobbles (dense to very dense, moist) (weathered till)		
		X	2				8	
					SM	Grades to gray (very dense, moist) (glacial till)		
		X	3				8	

Test pit completed at 4½ feet on 01/17/05
Slow groundwater seepage observed 2½ feet
No caving observed
Refusal at 4½ feet

5

10

Note: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

LOG OF TEST PIT TP-2

GeoEngineers

Project: Paine Field Ramp Expansion
Project Location: Everett, Washington
Project Number: 5530-003-00

Figure A-3
Sheet 1 of 1

DESCRIPTION

Surface Elevation (ft.): 588.5

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
0						SOD	3 inches sod
						SM	Brown silty fine sand with fine to coarse gravel and a trace of roots (loose, moist) (topsoil)
						SM	Brown silty fine to medium sand with occasional coarse sand, fine to coarse gravel and cobbles (very dense, moist) (weathered till)
	MD	8	102	68			
						SM	Gray silty fine sand with fine to coarse gravel, cobbles and occasional boulders (very dense, moist) (till)
	MD	7	109	50/4"			
				50/2"			
				50/5"			
				50/3"			
				50/5"			

Boring completed at 29.0 feet due to refusal on 06/10/96
No ground water encountered during drilling
Piezometer installed to 15.0 feet
No ground water measured in piezometer on 06/17/96

DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 8/2/96

Geo Engineers

LOG OF BORING

FIGURE A-3

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION Surface Elevation (ft.): 588.5
0						SOD	4 inches sod
						SM	Brown silty fine to medium sand with gravel (loose, moist) (topsoil)
						SM	Brown silty fine sand with fine to coarse gravel and cobbles (medium dense, moist) (weathered till)
	MD	9	133	28	■		
5						SM	Gray silty fine sand with fine to coarse gravel and cobbles (very dense, moist) (till)
				50/6"	■		
							Boring completed at 8.0 feet on 06/10/96 No ground water encountered during drilling

DEPTH IN FEET

0 5 10 15 20 25 30 35 40

Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 7/10/96

GeoEngineers

LOG OF BORING

FIGURE A-4

BORING B-3

Surface Elevation (ft.): 589.0

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
0						SOD	3 inches sod
						SM	Brown silty fine to medium sand with gravel (loose, moist) (topsoil)
						SM	Brown silty fine sand with fine to coarse gravel and cobbles (very dense, moist) (weathered till)
	MD	10	127	50/4"	■		
5						SM	Gray silty fine to medium sand with fine to coarse gravel, cobbles and occasional boulders (very dense, moist) (till)
				50/3"	■		

Boring completed at 8.8 feet on 06/10/96
No ground water encountered during drilling

DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 7/10/96

GeoEngineers

LOG OF BORING

FIGURE A-5

TEST DATA

BORING B-4

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION Surface Elevation (ft.): 588.5
					SOD	3 inches sod
					SM	Brown silty fine to medium sand with fine to coarse gravel and occasional cobbles (very dense, moist) (weathered till)
					SM	Gray silty fine to medium sand with fine to coarse gravel and occasional cobbles (very dense, moist) (till)

DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 7/10/96

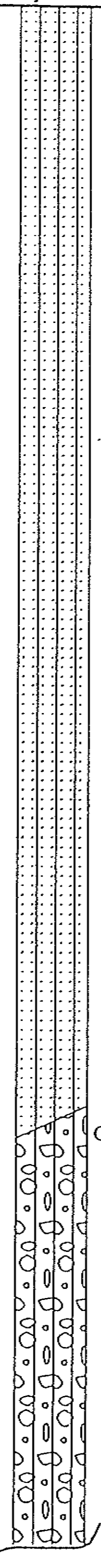
Geo Engineers

LOG OF BORING

FIGURE A-6

DESCRIPTION

Depth in Feet	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
40–69.5							
69.5–80						GM	Brown silty fine to coarse gravel with fine to coarse sand and occasional cobbles (very dense, wet)



Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 7/10/98

GeoEngineers

LOG OF BORING

FIGURE A-6

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
80							

Boring completed at 82.0 feet on 06/13/96
Ground water encountered at 69.0 feet during drilling
2-inch-diameter PVC casing installed to 74.0 feet

DEPTH IN FEET: 80, 85, 90, 95, 100, 105, 110, 115, 120

Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 7/10/96

Surface Elevation (ft.): 589.0

Depth in Feet	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
0						SOD	2 inches sod
						SM	Brown silty fine sand with fine to coarse gravel and cobbles (very dense, moist) (weathered till)
5	MD	7	108	60/5"	■		
15				50/5.5"	■		
						SM	Gray silty fine sand with fine to coarse gravel and cobbles (very dense, moist) (till)
25				50/6"	■		
35				50/6"	■		

Boring completed at 35.5 feet on 06/17/96
Ground water encountered at 15.0 feet during drilling
Piezometer installed to 17.0 feet

Note: See Figure A-2 for explanation of symbols

1468-018-01-1130 :DJM:JKT:WD 7/10/96

GeoEngineers

LOG OF BORING

FIGURE A-7

Appendix B

PIT TEST RESULTS & PHOTOS

PIT TEST REPORT:

Work at Paine Field was performed by Mr. Brian Osgood of AECOM and Clear Creek contractor's personnel on May 19th 2016. The purpose of the project was to perform a small-scale Pilot Infiltration Test (PIT) in accordance with Washington State Department of Ecology (WDOE) Stormwater Management Manual for Western Washington (SMMWW) (2012, Amended in December 2014). Relevant pages from WDOE SMMWW that describe the PIT test procedure are provided as Attachment A.

Work onsite began with marking the proposed test pit excavation at the PIT location to avoid utilities. Next, excavation began using a rubber tracked mini backhoe to create a 3-foot wide by 6-foot long by 3-foot deep test pit. Care was taken to keep excavated material from falling back into the excavation.

The trench corners were cleaned and squared up by hand with a shovel (Pictures 1 and 2). A vertical measuring device was placed in the center of the trench and water was pumped in (Picture 3); a timer was started for 6 hours at 09:40. Water was filled in the test pit to 2-foot bgs for the PIT and approximately 48 cubic feet (cf) of water was added to the test pit. The test pit walls were allowed to soak for 6 hours. A measuring rod was installed firmly in the center of the test pit for water level measurements.

After 6 hours, water in the test pit had dropped approximately 4 inches (Picture 4). The water was pumped out to attain the approximate 12-inch testing depth that is required for the PIT. The amount of water at the beginning of the test was approximately 18.5 cf. care was taken to expel the water as far away from the excavation as possible to mitigate water re-entering the excavation. The water level measurements were started in the test pit at 15:30 and were recorded every 15 minutes until the PIT was stopped at 16:30.

Shortly after the 1-hour water level measurements had commenced, rain started falling extremely hard, the crew took cover in vehicles for the remainder of the test, leaving the vehicles every 15 minutes to measure the water level (Pictures 5-8). It was observed that the water level had risen during the 1-hour test. This was most likely due to the heavy rain fall.

The amount of water at the end of the test was approximately 18.7 cf. Water remaining in the test pit was discharged to a location onsite. The test pit was backfilled and compacted with excavated soil.

RESULTS:

Infiltration rate was low enough that rainfall was able to fill the excavation. After reviewing rainfall data on WeatherUnderGround.com, approximately 0.16 inches of rain fell in during the hour when water level measurements were taken. The infiltration rate, before the correction factors were applied, was calculated to be 0.04 inches based on the water level measurements and rainfall data. After the correction factors that are provided in Table 3.3.1 of WDOE SMMWW Manual were applied, a final infiltration rate of 0.016 inches/hour was determined. Infiltration rate calculations are provided below.

Pilot Infiltration Test - Paine Field Passenger Terminal Expansion Building

Test Performed by: Brian Osgood
Date: May 19, 2016

Time (min)	Level "
0.00	12.50
15.00	12.7
30.00	
45.00	
60.00	

[redacted]

	Width	Length	Depth	Volume (Cu. In)	Cu. Ft	Gal	Drop Rate (In./Hr)
Total Excavation	36	72	32	82,944	48.00	359.09	N/A
Total Water	36	72	24.25	62,856	36.38	272.12	N/A
End of 6 hour soak	36	72	20.25	52,488	30.38	227.24	0.67
1 hour Test	36	72	12.35	32,011	18.53	138.59	N/A
End of Test	36	72	12.47	32,322	18.71	139.93	0.04
Rain Fall	0.16						

Infiltration Rate	
Total Area	2592
Height Change	(0.12)

Correction Factors	CFv	CFt	CFm
	0.9	0.5	0.9
Ksat-Design	0.016		

Table 3.3.1
Correction Factors to be Used With In-Situ Saturated Hydraulic Conductivity Measurements to Estimate Design Rates.

Issue	Partial Correction Factor
Site variability and number of locations tested	$CF_v = 0.33$ to 1.0
Test Method Large-scale PIT Small-scale PIT Other small-scale (e.g. Double ring, falling head) Grain Size Method	 $CF_t = 0.75$ $= 0.50$ $= 0.40$ $= 0.40$
Degree of influent control to prevent siltation and bio-buildup	$CF_m = 0.9$

Total Correction Factor, $CF_T = CF_v \times CF_t \times CF_m$

Table Sourced from WA Dept Ecology PIT rules 2014

Photos:

Picture 1:

Picture 2:

Picture 3:

Picture 4:

Picture 5:

Picture 6:

Picture 7:

Picture 8:

ATTACHMENT A

Relevant Pages from WDOE SMMWW Manual

Relevant pages from Washington State Department of Ecology (DOE) Stormwater Management Manual for Western Washington (SMMWW) that describe the PIT test procedure

Every 15-30 min, record the cumulative volume and instantaneous flow rate in gallons per minute necessary to maintain the water level at the same point on the measuring rod.

Keep adding water to the pit until one hour after the flow rate into the pit has stabilized (constant flow rate; a goal of 5% variation or less variation in the total flow) while maintaining the same pond water level. The total of the pre-soak time plus one hour after the flow rate has stabilized should be no less than 6 hours.

- After the flow rate has stabilized for at least one hour, turn off the water and record the rate of infiltration (the drop rate of the standing water) in inches per hour from the measuring rod data, until the pit is empty. Consider running this falling head phase of the test several times to estimate the dependency of infiltration rate with head.
- At the conclusion of testing, over-excavate the pit to see if the test water is mounded on shallow restrictive layers or if it has continued to flow deep into the subsurface. The depth of excavation varies depending on soil type and depth to hydraulic restricting layer, and is determined by the engineer or certified soils professional. Mounding is an indication that a mounding analysis is necessary.

Data Analysis

Calculate and record the saturated hydraulic conductivity rate in inches per hour in 30 minutes or one-hour increments until one hour after the flow has stabilized.

Note: Use statistical/trend analysis to obtain the hourly flow rate when the flow stabilizes. This would be the lowest hourly flow rate.

Apply appropriate correction factors to determine the site-specific design infiltration rate. See the discussion of correction factors for infiltration facilities in this Section 3.3, and the discussion of correction factors for bioretention facilities and permeable pavement in Section 3.4.

Example

The area of the bottom of the test pit is 8.5-ft. by 11.5-ft.

Water flow rate was measured and recorded at intervals ranging from 15 to 30 minutes throughout the test. Between 400 minutes and 1,000 minutes the flow rate stabilized between 10 and 12.5 gallons per minute or 600 to 750 gallons per hour, or an average of (9.8 + 12.3) / 2 = 11.1 inches per hour.

2. <u>Small-Scale Pilot Infiltration Test</u>

A smaller-scale PIT can be substituted for the large-scale PIT in any of the following instances.

- The drainage area to the infiltration site is less than 1 acre.

- The testing is for the LID BMP's of bioretention or permeable pavement that either serve small drainage areas and /or are widely dispersed throughout a project site.
- The site has a high infiltration rate, making a full-scale PIT difficult, and the site geotechnical investigation suggests uniform subsurface characteristics.

Infiltration Test

- Excavate the test pit to the estimated surface elevation of the proposed infiltration facility. In the case of bioretention, excavate to the estimated elevation at which the imported soil mix will lie on top of the underlying native soil. For permeable pavements, excavate to the elevation at which the imported subgrade materials, or the pavement itself, will contact the underlying native soil. If the native soils (road subgrade) will have to meet a minimum subgrade compaction requirement, compact the native soil to that requirement prior to testing. Note that the permeable pavement design guidance recommends compaction not exceed 90% - 92%. Finally, lay back the slopes sufficiently to avoid caving and erosion during the test. Alternatively, consider shoring the sides of the test pit.
- The horizontal surface area of the bottom of the test pit should be 12 to 32 square feet. It may be circular or rectangular, but accurately document the size and geometry of the test pit.
- Install a vertical measuring rod adequate to measure the ponded water depth and that is marked in half-inch increments in the center of the pit bottom.
- Use a rigid pipe with a splash plate on the bottom to convey water to the pit and reduce side-wall erosion or excessive disturbance of the pond bottom. Excessive erosion and bottom disturbance will result in clogging of the infiltration receptor and yield lower than actual infiltration rates. Use a 3-inch diameter pipe for pits on the smaller end of the recommended surface area, and a 4-inch pipe for pits on the larger end of the recommended surface area.
- Pre-soak period: Add water to the pit so that there is standing water for at least 6 hours. Maintain the pre-soak water level at least 12 inches above the bottom of the pit.
- At the end of the pre-soak period, add water to the pit at a rate that will maintain a 6-12 inch water level above the bottom of the pit over a full hour. The depth should not exceed the proposed maximum depth of water expected in the completed facility.
- Every 15 minutes, record the cumulative volume and instantaneous flow rate in gallons per minute necessary to maintain the water level at the same point (between 6 inches and 1 foot) on the measuring rod.

The specific depth should be the same as the maximum designed ponding depth (usually 6 – 12 inches).

- After one hour, turn off the water and record the rate of infiltration (the drop rate of the standing water) in inches per hour from the measuring rod data, until the pit is empty.
- A self-logging pressure sensor may also be used to determine water depth and drain-down.
- At the conclusion of testing, over-excavate the pit to see if the test water is mounded on shallow restrictive layers or if it has continued to flow deep into the subsurface. The depth of excavation varies depending on soil type and depth to hydraulic restricting layer, and is determined by the engineer or certified soils professional. The soils professional should judge whether a mounding analysis is necessary.

Data Analysis

See the explanation under the guidance for large-scale pilot infiltration tests.

3. Soil Grain Size Analysis Method

For each defined layer below the infiltration pond to a depth below the pond bottom of 2.5 times the maximum depth of water in the pond, but not less than 10 feet, estimate the initial saturated hydraulic conductivity (K_{sat}) in cm/sec using the following relationship (see Massmann 2003, and Massmann et al., 2003). For large infiltration facilities serving drainage areas of 10 acres or more, soil grain size analyses should be performed on layers up to 50 feet deep (or no more than 10 feet below the water table).

$$\log_{10}(K_{sat}) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{\text{fines}} \qquad (1)$$

Where, D_{10}, D_{60} and D_{90} are the grain sizes in mm for which 10 percent, 60 percent and 90 percent of the sample is more fine and f_{fines} is the fraction of the soil (by weight) that passes the number-200 sieve (K_{sat} is in cm/s).

For bioretention facilities, analyze each defined layer below the top of the final bioretention area subgrade to a depth of at least 3 times the maximum ponding depth, but not less than 3 feet (1 meter). For permeable pavement, analyze for each defined layer below the top of the final subgrade to a depth of at least 3 times the maximum ponding depth within the base course, but not less than 3 feet (1 meter).

If the licensed professional conducting the investigation determines that deeper layers will influence the rate of infiltration for the facility, soil layers at greater depths must be considered when assessing the site's hydraulic conductivity characteristics. Massmann (2003) indicates that where the water table is deep, soil or rock strata up to 100 feet below an infiltration facility can influence the rate of infiltration. Note that only the layers near and above the water table or low permeability zone (e.g., a clay, dense glacial till, or rock layer) need to be considered, as the layers below the ground water table or low permeability zone do not significantly influence the rate of infiltration. Also note that this equation for estimating K_{sat} assumes minimal compaction consistent with the use of tracked (i.e., low to moderate ground pressure) excavation equipment.

If the soil layer being characterized has been exposed to heavy compaction (e.g., due to heavy equipment with narrow tracks, narrow tires, or large lugged, high pressure tires) the hydraulic conductivity for the layer could be approximately an order of magnitude less than what would be estimated based on grain size characteristics alone (Pitt, 2003). In such cases, compaction effects must be taken into account when estimating hydraulic conductivity.

For clean, uniformly graded sands and gravels, the reduction in K_{sat} due to compaction will be much less than an order of magnitude. For well-graded sands and gravels with moderate to high silt content, the reduction in K_{sat} will be close to an order of magnitude. For soils that contain clay, the reduction in K_{sat} could be greater than an order of magnitude.

If greater certainty is desired, the in-situ saturated conductivity of a specific layer can be obtained through the use of a pilot infiltration test (PIT). Note that these field tests generally provide a K_{sat} combined with a hydraulic gradient (i.e., Equation 5 in Section 3.3.8). In some of these tests, the hydraulic gradient may be close to 1.0; therefore, in effect, the test infiltration rate result is the same as the hydraulic conductivity. In other cases, the hydraulic gradient may be close to the gradient that is likely to occur in the full-scale infiltration facility. The hydraulic gradient will need to be evaluated on a case-by-case basis when interpreting the results of field tests. It is important to recognize that the gradient in the test may not be the same as the gradient likely to occur in the full-scale infiltration facility in the long-term (i.e., when ground water mounding is fully developed).

Once the K_{sat} for each layer has been identified, determine the effective average K_{sat} below the pond. K_{sat} estimates from different layers can be combined using the harmonic mean:

$$K_{equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \qquad (2)$$

Where, d is the total depth of the soil column, d_i is the thickness of layer "i" in the soil column, and K_i is the saturated hydraulic conductivity of layer "*i*" in the soil column. The depth of the soil column, *d*, typically would include all layers between the pond bottom and the water table. However, for sites with very deep water tables (>100 feet) where ground water mounding to the base of the pond is not likely to occur, it is recommended that the total depth of the soil column in Equation 2 be limited to approximately 20 times the depth of pond, but not more than 50 feet. This is to ensure that the most important and relevant layers are included in the hydraulic conductivity calculations. Deep layers that are not likely to affect the infiltration rate near the pond bottom should not be included in Equation 2.

Equation 2 may over-estimate the effective K_{sat} value at sites with low conductivity layers immediately beneath the infiltration pond. For sites where the lowest conductivity layer is within five feet of the base of the pond, it is suggested that this lowest K_{sat} value be used as the equivalent hydraulic conductivity rather than the value from Equation 2. Using the layer with the lowest K_{sat} is advised for designing bioretention facilities or permeable pavements. The harmonic mean given by Equation 2 is the appropriate effective hydraulic conductivity for flow that is perpendicular to stratigraphic layers, and will produce conservative results when flow has a significant horizontal component such as could occur due to ground water mounding.

Correction Factors

Correction Factors for PIT results and Grain Size Method - The K_{sat} obtained from the PIT test or Grain Size Method is a measured (initial) rate. This measured rate must be reduced through correction factors that are appropriate for the design situation to produce a design infiltration rate. This adjustment is made in Step 5 of the Design of Infiltration Facilities (Section 3.3.4).

Correction factors account for site variability, number of tests conducted, uncertainty of the test method, and the potential for long-term clogging due to siltation and bio-buildup. Table 3.3.1 summarizes the typical range of correction factors to account for these issues. The specific correction factors used shall be determined based on the professional judgment of the licensed engineer or other site professional considering all issues that may affect the infiltration rate over the long term, subject to the approval of the local jurisdictional authority**.**

Table 3.3.1 Correction Factors to be Used With In-Situ Saturated Hydraulic Conductivity Measurements to Estimate Design Rates.	
Issue	**Partial Correction Factor**
Site variability and number of locations tested	CF_v = 0.33 to 1.0
Test Method Large-scale PIT Small-scale PIT Other small-scale (e.g. Double ring, falling head) Grain Size Method	 CF_t = 0.75 = 0.50 = 0.40 = 0.40
Degree of influent control to prevent siltation and bio-buildup	CF_m = 0.9

Total Correction Factor, $CF_T = CF_v \times CF_t \times CF_m$

CF_T is used in step 5 of the Design of Infiltration Facilities (Section 3.3.4) to adjust the measured (initial) saturated hydraulic conductivity.

K_{sat} design = K_{sat} initial X CF_T

Site variability and number of locations tested (CF_v) - The number of locations tested must be capable of producing a picture of the subsurface conditions that fully represents the conditions throughout the facility site. The partial correction factor used for this issue depends on the level of uncertainty that adverse subsurface conditions may occur. If the range of uncertainty is low - for example, conditions are known to be uniform through previous exploration and site geological factors - one pilot infiltration test (or grain size analysis location) may be adequate to justify a partial correction factor at the high end of the range.

If the level of uncertainty is high, a partial correction factor near the low end of the range may be appropriate. This might be the case where the site conditions are highly variable due to conditions such as a deposit of ancient landslide debris, or buried stream channels. In these cases, even with many explorations and several pilot infiltration tests (or several grain size test locations), the level of uncertainty may still be high.

A partial correction factor near the low end of the range could be assigned where conditions have a more typical variability, but few explorations and only one pilot infiltration test (or one grain size analysis location) is conducted. That is, the number of explorations and tests conducted do not match the degree of site variability anticipated.

Uncertainty of test method (CF_t) accounts for uncertainties in the testing methods. For the full scale PIT method, CF_t = 0.75; for the small-scale PIT method, CF_t = 0.50; for smaller-scale infiltration tests such as the double-ring infiltrometer test, CF_t = 0.40; for grain size analysis, CF_t =